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Centrifuge modelling of monopiles in dense sand at The Technical University of Denmark

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Abstract: The pile-soil interaction is of great importance in the design of monopiles supporting offshore wind turbines. Especially the predication of accumulation of rotations, change in stiffness and damping is difficult. The design tools used today cannot in a proper way take the effects from cyclic, lateral loading into account. In order to develop simple design tools for cyclic lateral loading, a large centrifuge test series is ongoing at the Technical University of Denmark (DTU). This paper presents the main conclusions found from centrifuge testing over the last 4 years and gives indication where research is going. From monotonic tests “modelling of models” has been performed to identify scaling errors. It appears that the non-linear stress distributions occurring in centrifuge tests should be taken into account when dealing with stiff monopiles. Also the effect from load eccentricity has been investigated and initial results indicate that the normalised initial stiffness is unaffected but the ultimate capacity is increased by increasing load eccentricity. Accumulation of rotations and increase in secant stiffness has been seen from cyclic, load-controlled tests. One initial test has been used to generate cyclic p-y curves and is here presented. This is the first step in the development of a cyclic spring element which can be used as a simple engineering tool in the prediction of accumulation of rotations, change in stiffness and damping.

Keywords: Centrifuge modeling, Monopiles, Monotonic & Cyclic loading, Sand, Renewable energy.

1 INTRODUCTION

Monopiles are today one of the most popular foundation methods for offshore wind turbines. These piles are often installed in dense sand at water depths ranging from 5-30 meters. A monopile is a single large diameter tubular steel pile driven 5 to 6 times its diameter into the seabed. The diameter of the piles ranges from 4-6 meters. Monopiles for wind turbines are affected by lateral loads from waves and winds, which subject the pile at seabed level with shear forces and moments corresponding to the load eccentricity, (Byrne & Houlsby 2003).

Today the design of monopiles is carried out by modelling the pile as a beam and the soil as a system of uncoupled non-linear springs, (API 2007). This method has successfully been used in pile design for offshore oil and gas platforms. The design methodology originates from tests on long slender piles with a small load eccentricity, (Reese & Matlock 1956) & (McClelland & Focht 1956). Even though this methodology was originally calibrated to slender piles, it is today used for design of large diameter stiff monopiles.

If the monopile foundation concept shall succeed in larger water depths, the methodology has to be improved. Therefore investigations with a special focus on lateral loading of monopiles supporting wind turbine are carried out. In the design of these piles especially the accumulation of rotations, the overall pile-soil stiffness and the damping of the soil are key parameters.

The first large wind farm, Horns Rev 1, using monopiles was established in 2002 and the research field for these monopiles is therefore also new. Different investigations on monopiles supporting wind turbines have though been performed to improve the current design methodology. This paper only deals with monopiles installed in sand and to give a short overview, some of the contributions are described here.

The presented research is summarized in Table 1. First of all it seen that the original tests, exemplified by (Cox, Reese & Grubbs 1974), are performed on long slender piles. Monopiles for wind turbines are short and stiff piles. The monopile research presented here is therefore concentrated on rigid piles ranging from 4 - 6 d penetration.

Table 1. Summary of the research

Authors	Model	Prototype diameter d [m]	Load eccentricity e/d	Pile penetration L/d	Number of Cycles N [-]
(LeBlanc, Houlsby & Byrne 2010)	1g	4	4	5.4	60000
(Cuéllar, Baeßler & Rücker 2009)	1g	7.5	4	4	5000000
(Li, Haigh & Bolton 2010)	Ng	5	14.4	5	1000
(Klinkvort, Hededal & Svensson 2011)	Ng	2	15	6	500
(Achmus, Kuo & Abdel-Rahman 2009)	num	5	2.6	2.6-5.3	10000
(Zania & Hededal 2011)	num	2	15	6	Monotonic
(Cox, Reese & Grubbs 1974)	Full scale	0.6	4.6	35	25
(Hald et al. 2009)	Full scale	4	17.5	5.5	Monotonic

In order to investigated the pile soil interaction different approaches can be taken. In principle the approaches can be divided into three groups; Investigations on a scaled model, Investigation on a full-scale model or investigation on a numerical model. The different methodologies, also presented in Table 1, all have their advantages.

The 1g experiments are relative easy to access and instrument, and series with many load cycles can therefore be carried out. The stress distribution in a 1g experiment is not identical with full-scale condition and scaling to prototype is therefore difficult. Centrifuge experiment at Ng is carried out at a stress level corresponding to prototype. This makes the scaling to prototype easier but still scaling laws should be used with care. Here it is though difficult to perform test which takes longer than one working day. This gives a limitation of the number of cycles. When using models in reduced scale the soil sample can be created artificial and the soil which is used in the test can therefore by quantified in a high degree. The big advantage of the numerical model is that they allow you to investigate stress paths, soil-pile interaction etc. in more details, but you have to have a constitutive law which is reliable. Full scale testing is of course a preferable method in investigations. The price of such testing is high, and also the interpretations of soil conditions and loads can be a challenge.

2 EXPERIMENTAL METHODOLOGY

In order to understand the behaviour of rigid monopiles, physical modelling in a centrifuge and numerical modelling are used at the Technical University of Denmark. It is important to recognize pro and cons with a given methodology, and we believe that using different methodologies is the best way to investigate a given problem and together these two methodologies give a strong investigation tool. This paper deals only with the centrifuge modeling of monopiles.

For general centrifuge modelling theory see e.g. (Schofield 1980).

2.1 Centrifuge setup at DTU

The centrifuge facility at DTU was constructed in 1976 and has been upgraded over the years. A sketch of the present setup can be seen in Figure 1. The capacity of the beam centrifuge is approximately 100 g ton and is capable of providing an artificial gravitation of around 90g. The arm of the centrifuge is 2.63 m. Presently the research at DTU is focusing on monopiles and the test setup has been changed to be suitable for large diameter piles loaded with a large load eccentricity. This setup can perform tests on offshore monopiles with an effective stress distribution identical to 5 meter in diameter.

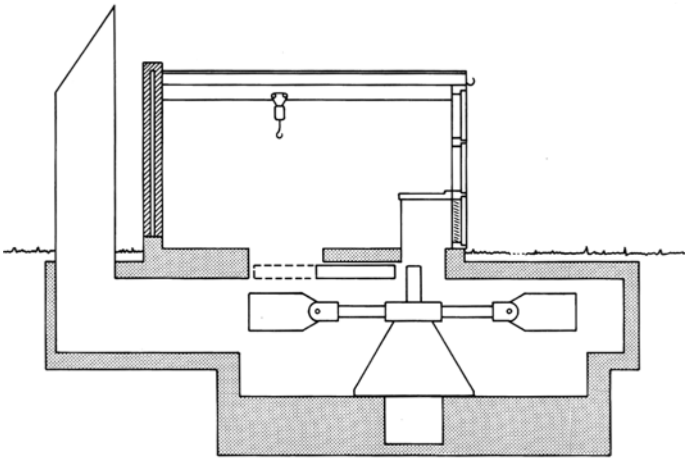

Figure 1. The geotechnical beam centrifuge at DTU, (Fuglsang & Nielsen 1988).

Monopiles can be installed in-flight, but there is a limitation of 20kN limit on the jack. The lateral capacity is affected by the installation, and installation at full stress level is therefore preferable, this was also shown by (Dyson & Randolph 2001). The procedure for a test sequence is to spin the centrifuge up and install the pile. Then the centrifuge has to be stopped to remove the jack and then mount the lateral loading equipment. Afterwards the centrifuge is accelerated again, to a given soil stress level and the lateral test is performed.

The DAQ hardware samples the signals from the measurement sources and digitizes the signals for storage, analysis, and presentation. All these devices are located on the centrifuge and connected to a flight computer. The flight computer on the centrifuge can be controlled by a "Remote Desktop Connection" from the control computer in the laboratory via a wireless internet connection, (Klinkvort 2010).

Solid steel piles without instrumentations have been used in preliminary tests. The results from these piles will be presented in this paper.

Two piles with strain-gauges have been constructed; one solid steel pile with a diameter of 24mm with five strain-gauge levels and a 2mm epoxy coating leading to a total diameter of 28mm and one solid steel pile with a diameter of 36 mm with 10 strain-gauges levels and protected by a 2mm epoxy coating leading to a total model pile diameter of 40mm. The choice of using strain-gauge instead of direct contact measuring is done to capture the integrate response of the pile –soil interaction. In this way the active, passive and friction pressure of the pile is integrated into one soil resistance. This integrated soil resistance is the one normally used in pile design using p-y curves.

The stiffness of these model piles is much stiffer than used for offshore wind turbines. Numerical investigations at DTU by (Zania & Hededal 2011) have showed that an increase in stiffness of a rigid pile does not change the response. Also (Dyson, Randolph 2001) showed that the lateral response from an open ended and a closed ended pile was identical in calcareous sand. It is therefore believed that results from the solid strain-gauge mounted pile behave as a monopile supporting a wind turbine.

3 RESULTS

The centrifuge experiments were carried out in Fontainebleau sand which is uniform silica sand from France which consists of fine and rounded particles. The classification data can be seen in Table 1. The sand is prepared in the centrifuge container by dry pluvation. The centrifuge at DTU uses a spot pouring hopper (SPH) for the preparation of the sand sample. Due to the circular geometry of the container and pile, the sand is prepared using a circular travelling loop. The sand is installed in a container with a inner diameter of 50 cm and a height of 49 cm. CPT tests have been carried out to validate the pouring method. All these CPT tests showed the soil sample has a good homogeneity in the container, (Leth 2011). An average relative density is then calculated from the weight of the sand sample. A relative density of the tests samples is sought to be 0.9 which is reflecting typical offshore conditions in the North Sea. With the given pluvation technique this goal is reached within 5 % for all tests.

Table 1. Soil parameters Fontainebleau sand (Leth, Krogsbøll & Hededal 2008).

Parameter	Value	Dimension
Density grains, G_s	2,646	kg/m^3
Average grain size, d_{50}	180	μm
Min void ratio e_{min}	0.548	
Max void ratio e_{max}	0.859	
Coefficient of uniformity C_u	1.6	

3.1 Monotonic tests

As a first step in understanding the monopile behavior, monotonic tests have been performed, (Leth, Krogsbøll & Hededal 2008), (Klinkvort 2010), (Klinkvort & Hededal 2010), (Klinkvort, Leth & Hededal 2010), (Leth 2011) & (Klinkvort, Hededal & Svensson 2011). These tests are also used in the cyclic analyses to see how close to the capacity the cyclic test is performed. In the next section some of the findings on monotonic testing from DTU will be presented.

3.1.1 Modeling of models

In order to investigate the scaling laws modeling of models have been performed. Five model monopiles with five different diameters was subjected to a stress field which for all the pile was identical with a 1 meter in diameter pile. The result from this investigation is shown in Figure 6. Here it can be seen that the response from these five piles not are identical as expected. Special the smallest pile has a different response. It is though clearly seen that the largest pile have the smallest stiffness and bearing capacity, The initial stiffness and the maximum capacity is increasing with decreasing diameter.

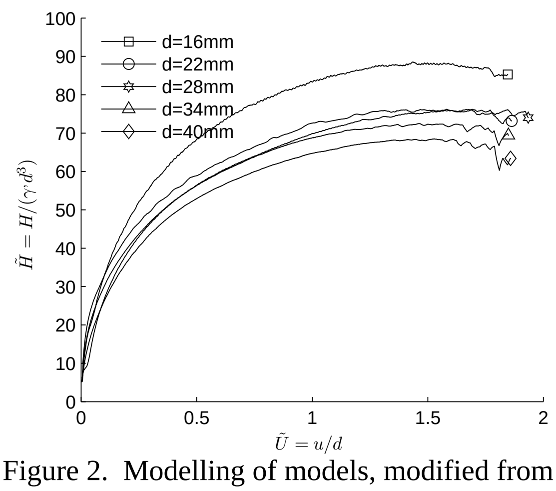

Figure 2. Modelling of models, modified from (Klinkvort & Hededal 2010).

One explanation of this could be the non-linear stress error effect. The increase in gravity is depending on the rotating arm. This arm is increasing down through the soil sample. This leads to a parabolic shaped stress distribution, in contrast to the linear increase seen in prototype. The tests are designed so the soil stress is too small in the upper part at the soil sample and too large in the bottom of the sample. In the depth of 2/3 of the pile penetration identical stress with prototype is achieved. This is done to minimize the stress error, (Taylor 1995). The larger diameter model piles is longer than the small model piles, this will introduce a larger stress error. When interpreting centrifuge data it is therefore important to recognise this stress level scaling error and to take the non-linear stress distribution into account. This assumption has to be validated.

3.1.2 Effect from load eccentricity

The loads acting on an offshore wind turbine is coming from wind and waves. This will load the monopile support with a shear force and bending moment at sea bottom. The ratio between shear and moment will vary dependent on wind and waves. The ratio between shear and moment is also a measure of the resultant load eccentricity. In order to investigate the effect from changing the load eccentricity tests with three different load eccentricities was carried out. These tests were presented in (Klinkvort, Leth & Hededal 2010). The total response of the pile was measured and the conclusion was that changing the load eccentricity leads to a different failure mechanism in the soil. In Figure 3 the results from three tests on a pile with 6d penetration and different load eccentricities are shown. The results are normalized with in-flight soil density and the diameter of the pile. On the left hand side the normalized load is plotted against the normalized displacement. Here it can be seen that the pile with a low eccentricity here e=2.5d have a different response from that of the piles with a higher eccentricity. This is investigated further; on the right hand side the same test is shown. Here is the normalized moment plotted against the deflection. Here it can be seen that the initial stiffness for the pile are only controlled by the applied bending moment whereas it seems that the pile with a high load eccentricity has a higher relative bearing capacity that the piles with lower eccentricities.

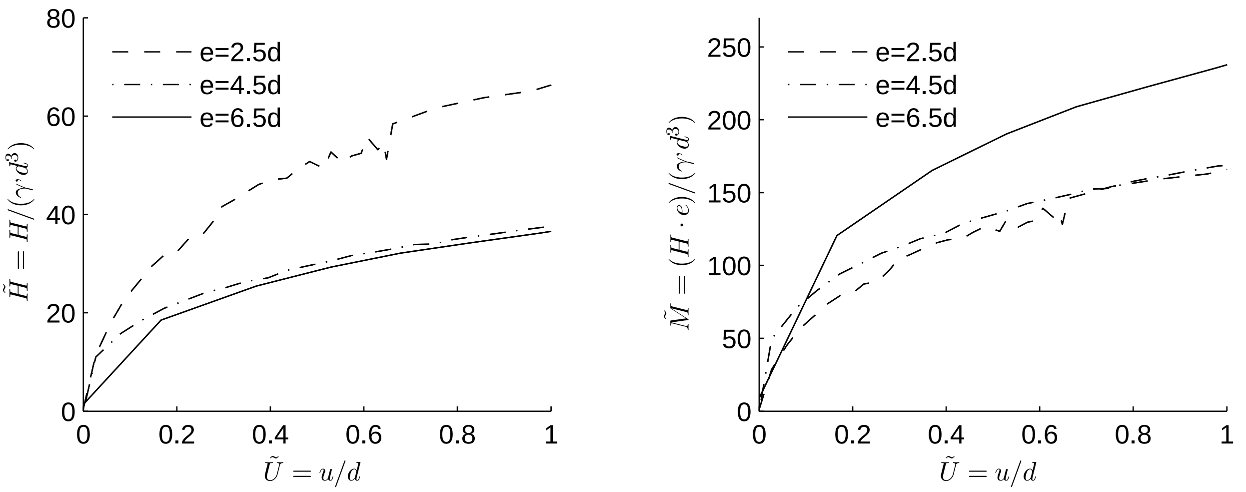

Figure 3. **Left:** Load vs. Deflection response, (Klinkvort, Leth & Hededal 2010). **Right:** Moment vs. Deflection response, modified from

3.2 Cyclic tests

Several cyclic tests have been performed at DTU and are described in (Klinkvort 2010), (Klinkvort & Hededal 2010), (Klinkvort, Leth & Hededal 2010), (Leth 2011) & (Klinkvort, Hededal & Svensson 2011). Common for these tests is they are load controlled tests in dry sand. The number of cycles the piles has been subjected for have been ranging from 100 to 500. For all performed test accumulation of displacement/rotation have been seen and also an increase in secant stiffness. Here is given one example of the interpretation of such a test.

A set of non-dimensional parameters are used to describe the applied cyclic loads. This approach is identical to the one chosen by (LeBlanc, Houlsby & Byrne 2010).

$$\zeta_b = \frac{P_{\max}}{P_{mon}} \qquad \zeta_c = \frac{P_{\min}}{P_{\max}} \tag{1}$$

Here, P_{mon} is the maximum capacity found from a monotonic test, P_{min} is the minimum load in cyclic loading and P_{max} is the maximum load in cyclic loading. The value ζ_{b} is thus a measure of how close the cyclic loading is to the maximum load capacity, and ζ_{c} is defining the characteristic of the cyclic loading. From these non-dimensional parameters a test program can be designed. The maximum capacity is found from the monotonic test. The cyclic loading is carried out using a feedback control system. A sinusoidal signal is generated according to the non-dimensional parameters wanted. Due to the feedback control system it is not always that the pile response is exactly as wanted. This can lead to difference between the measured non-dimensional parameters and the planned.

In Figure 4 an example of a cyclic test series is shown. A small difference in maximum and minimum values can be seen. This is due to difficulties in the feedback control. From a test like this maxima and minima from every cycle can be found.

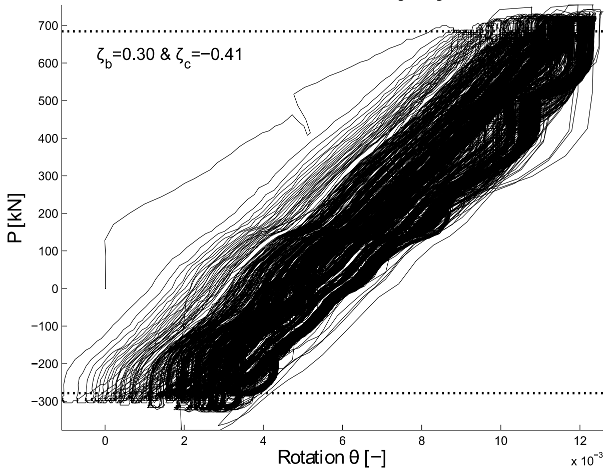

Figure 4. Load deflection result for cyclic test T3, (Klinkvort, Hededal & Svensson 2011).

When the extremes are found, deflection/ rotation and secant stiffness from every cycle can be determined. The accumulation of rotation may be fitted to a power function, see equation (3).

$$\theta_{max,N} = \theta_{max,1} \cdot N^{\alpha} \tag{2}$$

The change in secant stiffness can be described as:

$$k_{\mathrm{sec.},N} = K_0 + \kappa \ln(N) \tag{3}$$

To give an example here the rotation and stiffness is plotted against the number of cycle. Some scatter in the data can be seen, but the power and logarithmic functions seems to capture the accumulation of rotation and change in secant stiffness quite well, see Figure 5. This is the case for all the cyclic tests performed.

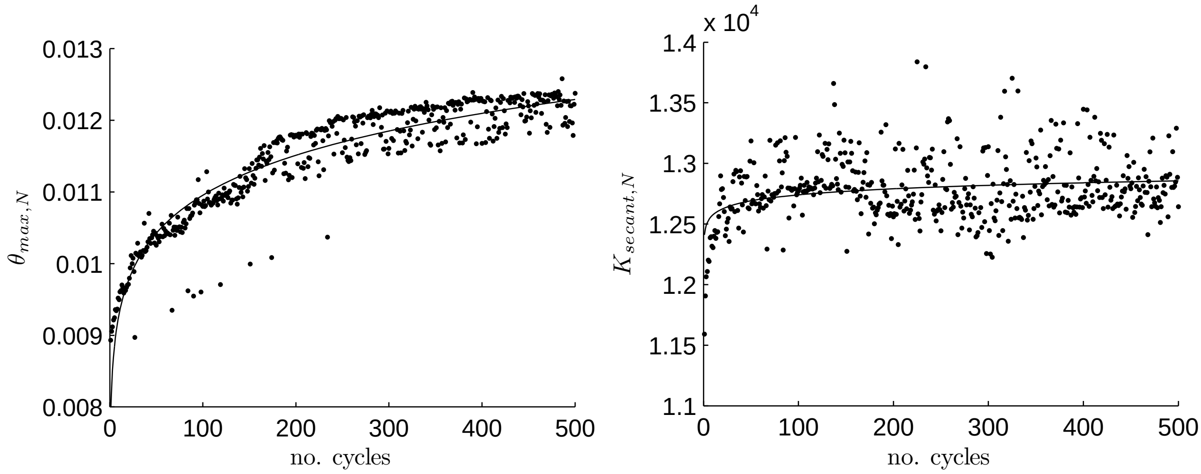

Figure 5. Maximum rotation and secant stiffness from every cycle modified from, (Svensson 2010).

From Figure 5, it can be seen that an asymptotic value of the rotation is not reached. Hence, tests with an even larger number of cycles should be carried out in order to get a more reliable estimate on the accumulation law.

3.3 Cyclic p-y curves

At the moment only results on the total response of centrifuge tested monopiles have been published and shown in this paper. To get a better understanding of the pile soil interaction, tests with strain-gauge mounted on the piles are ongoing now. As a first attempt to generate cyclic p-y curves from the moment distribution of the pile, a pile with five strain-gauge levels was constructed. From the tests performed on this pile it was possible to generate cyclic p-y curves. In Figure 6 cyclic curve from a strain-gauge level 1 d in the soil is shown.

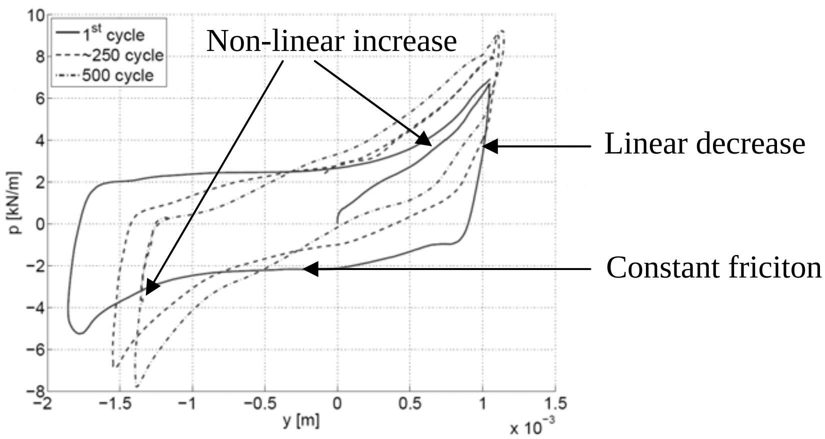

Figure 6. Cyclic p-y curve, (Svensson 2010).

When the pile is loaded the soil resist the pile movement and the soil pressure will build up. When the load is reversed the soil resistance will drop, then the pile is subjected to a more or less constant force until it reach a point where the resistance increases again, when the load is reversed again the soil will drop and so on. This is interpreted as build up of resistance when the pile “pushes the soil”, when the pile is loaded into the other direction the load will drop linearly. Then the pile will move in an area where only friction is loading the pile with a constant force. Until the pile again hits the soil face and starts to build resistance up again. This approach has been defined in a cyclic spring element which can be use in a cyclic Winkler analysis of the pile. The cyclic spring is presented in (Hededal & Klinkvort 2010).

This cyclic p-y behavior is at the moment one of the focus points in the research. A pile with 10 strain–gauge levels is used to generate p-y curves and results from this pile are used to improve the description of the cyclic spring-element. The goal is to have a good robust cyclic spring element, which when it is used in a Winkler model predicts the total accumulation of rotations, the change in secant stiffness and the damping of the soil.

4 DISCUSSION

In this paper different examples of monotonic and cyclic lateral load test have been described. All tests have been conducted in the Geotechnical beam centrifuge at DTU. One common factor for all the presented tests is that they have been performed in dense dry sand. The sand located offshore where monopiles are installed, is dense sand but it is not dry. This means the condition for a real monopile could be drained, partial drained and maybe undrained even for sand. It is important to understand that the tests presented here only represent the fully drained case.

The next step in the research at DTU is to add water to the soil sample. This enables the possibility of performing test in saturated sand, corresponding to offshore conditions. This will capture if there is any effect of water flowing through the soil even though it is fully drained, but the drainage is not scaled correctly in this way.

If an investigation of the possibly accumulation of pore pressure has to be investigated it is necessary to scale the loading frequency and the fluid viscosity with N. This has to be done to have full similarity between model and prototype. The fluid viscosity can be achieved by a mixture of water and metolose. The loading frequency has to be scaled as well and with the given setup it is not possible to apply loads with a frequency in the range of 25 Hz which is needed.

The number of cycles is another issue which has to be taken into account. The cyclic tests presented here have a total number of cycles 500. This is much more than the number of cycles from the original tests but the number still has to be increased.

5 CONCLUSION

This paper has presented an overview of the centrifuge research at DTU of monopile foundation supporting offshore wind turbines. From monotonic test the scaling laws has been investigated through the concept of modeling of models. These tests indicate that the non-linear stress distribution which introduces a scaling error should be taken into account. It was not possible to generate identical responses as expected. This issue has to be investigated in more detail. Also from monotonic test the effect from load eccentricity has been investigated, and a change in failure mechanism was indicated. It seems like the initial stiffness of the pile is un-affected by the load eccentricity were as the ultimate capacity is increasing by increasing the load eccentricity. This effect can be investigated in more details with a strain-gauge mounted pile which is ongoing at the moment. In practice this may be of importance for offshore wind turbines subjected to a combination of wind load from the rotor acting 60–100 m above seabed level and wave forces acting relatively closer to the seabed.

From all the cyclic tests carried out, accumulations of deflections were seen. The secant stiffness of every cycle was measured revealing that the cyclic loading led to an increase in secant stiffness. From the centrifuge tests it was clearly seen that no reduction of the bearing capacity of dry sand occurs due to cyclic loading.

One initial test showing a cyclic p-y curve where presented. A simple interpretation of this test was given. This interpretation is used to construct a cyclic-spring element which can be used in a Winkler analysis. The goal is to develop a good robust cyclic spring element, which when it is used in a Winkler model predicts the total accumulation of rotations, the change in secant stiffness and the damping of the soil.

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